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COMPILATION OF BENCHMARK LATERAL LOAD TESTS ON PILES IN SAND AND CLAY

An Investigation Conducted by

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13. ABSTRACT *(Maximum 200 words)*

This report presents a compilation of 14 lateral load tests on piles in sands and clays. It serves as a benchmark for comparison of tests results to analysis. Each case study has pile section properties and soil characteristics. Each set of data contains the loading applied and the measured response.

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CASE 1

Reference: Speer, D., "Shaft Lateral Load Test Terminal Separation," Unpublished report, California Department of Transportation, 1992.

Data on piles:

Kind: Reinforced concrete (drilled shafts)

Diameter: 88.5 in., Shaft A and Shaft B

Reinforcement: 28 No. 14 reinforcing bars; diameter to center of cage is 74.3 in.

Properties of materials: f_c' of concrete is 5,000 psi; f_y of steel is 71,900 psi

Penetration: 40.85 ft., Shaft A; 45.15 ft., Shaft B

Data on subsurface materiall

The subsurface material was a sandstone and, as is often the case, secondary structure was prominent. The sandstone was found to be medium to fine-grained (with grain sizes from 0.10 to 0.50 mm), well sorted and thinly bedded (with thicknesses from 25 to 75 mm). In most of the corings, the sandstone was described as intensely to moderately fractured with bedding joints, joints, and fracture zones.

With regard to the technique used in making the borings, the following statement was made: "After bedrock was encountered, sampling was continued using a NWD4 core barrel in a 4 inch diameter

cased hole. A three and seven eight inch tricone rock bit was used to advance the casing and clean the borehole." Three borings (one very short) were made and recovery was 100% except for two cores of 94% and 60% near the ground surface in one of the borings.

The *RQD* ranged from 0 to 80 and averaged 45 for the two main borings. No compression tests of intact specimens were reported.

Pressuremeter tests were performed at the site, but only the early part of the curve was obtained. The reported values from the pressuremeter tests are shown in three pages that follow

Loading

The load was applied in increments to both piles simultaneously. The axes of the piles were 50 ft apart. High-strength steel bars were threaded through cast-in-place pipes and the loads were applied with hydraulic cylinders. The load was applied at 49.45 in. above the rock line for Pile A and 50.75 in. for Pile B.

Instrumentation

A number of types of instruments were employed. The principal data that are analyzed are the readings of mechanical transducers that measured pile-head deflection and the corresponding loads that were measured by load cells. Data from instruments that were used to obtain bending moments and lateral pressures were not analyzed because calibration curves were unavailable.

Pile-Head Deflection as a Function of Applied Load

Numerical values of deflection versus load are shown in a table that follows. As noted, the deflections were measured fairly close to he rock line: 1.44 in. for Pile A, and 3.84 in. for Pile B. For loads greater than 500 kips, Pile B showed somewhat larger deflections than did Pile A. Furthermore, the relatively large increase in deflection for Pile B for the 100-kip increment of load from 1900 to 2000 kips suggests that Pile B was in the failure condition. The assumption can reasonably be made that a plastic hinge was developed in Pile B at the maximum load of 2000 kips.

DATA FROM TESTS WITH PRESSUREMETER, SAN FRANCISCO						
SHAFT LATERAL LOAD TEST TERMINAL SEPARATION						
Boring B-3						
B-3	Depth, ft	Gm, psi	Em, psi	Poh, psi		
A	23.7	6,600	17,500	155		
B	25.5	5,500	14,700	60		
C	29.9	7,900	20,900	46		
D	35.0	64,000	170,200	130		
E	39.8	76,600	203,900	365		
F	45.0	26,700	70,900	362		
G	54.5	98,800	262,700	332		
Elevation at top of boring was 96.1; pit was excavated to top of rock						
at an average elevation of 76.25; therefore, overburden of soil was						
19.85 feet. Thus, Test A was performed at 3.85 ft below the top of						
rock in the test pit.						
Gm = shear modulus						
Em = pressuremeter modulus (Poisson's ratio = 0.33)						
Poh = horizontal total stress at rest						

DATA FROM TESTS WITH PRESSUREMETER, SAN FRANCISCO						
SHAFT LATERAL LOAD TEST TERMINAL SEPARATION						
Boring B-2						
B-2	Depth, ft	Gm, psi	Em, psi	Poh, psi		
A	16.9	23,100	61,600	175		
B	24.0	14,300	38,100	190		
Elevation at top of bo	ing was 96.5. The pit was excavated to top of rock					
at an average elevation of 76.25; therefore, overburden of soil was						
20.25 feet. Thus, Test B was at 3.75 feet below top of						
rock in the test pit.						
Gm = shear modulus						
Em = pressuremeter modulus (Poisson's ratio = 0.33)						
Poh = horizontal total stress at rest						

DATA FROM TESTS WITH PRESSUREMETER, SAN FRANCISCO						
SHAFT LATERAL LOAD TEST TERMINAL SEPARATION						
Boring B-1						
B-1	Depth, ft	Gm, psi	Em, psi	Poh, psi		
A	22.0	31,300	83,200	190		
B	26.7	3,800	10,100	46		
C	31.5	7,700	20,400	70		
D	36.5	20,200	53,700	95		
E	38.5	9,700	25,700	51		
F	42.7	17,500	46,600	100		
G	47.3	31,700	84,400	139		
H	53.0	76,500	203,500	378		
I	57.0	93,500	248,700	373		
Elevation at top of boring was 96.3; pit was excavated to top of rock						
at an average elevation of 76.25; therefore, overburden of soil was						
20.05 feet. Thus, Test A was performed at 1.95 ft below the top of						
rock in the test pit.						
Gm = shear modulus						
Em = pressuremeter modulus (Poisson's ratio = 0.33)						
Poh = horizontal total stress at rest						

Lateral-load testing of drilled shafts in rock at San Francisco							
Lateral load, kips		Shaft A			Shaft B		
		Deflection	at 1.44 in, in		Deflection	at 3.84 in., in	
0		0			0		
100		0.0017			0		
200		0.0027			0		
300		0.004			0		
400		0.0084			0.0055		
500		0.0158			0.01657		
600		0.0303			0.0442		
700		0.0471			0.0718		
800		0.0774			0.1022		
900		0.1027			0.1326		
1000		0.1346			0.1796		
1100		0.1683			0.2238		
1200		0.2053			0.2845		
1300		0.2423			0.3425		
1400		0.2827			0.4116		
1500		0.3298			0.4696		
1600		0.3803			0.5525		
1700		0.4476			0.663		
1800		0.5116			0.7956		
1900		0.6428			1.05		
2000					1.945		

Bagnolet (Case 2)

Kerisel (1965)[1] reported the results of three, short-term, static lateral-load tests of a closed-ended "bulkhead caisson". The cross section of the pile is shown in Fig. 1. Two sheet-pile sections were welded together to form the pile. The three tests were performed on the same pile which was recovered and reinstalled for all tests following the first one. The bending stiffness *EI* was given as 25,500 kN-m^2 (8,886,000 kip-in^2) and if *E* is selected as 200,000 MPa (29,000 kip/in^2) the value of *I* is 0.0001275 m^4 (306.4 in^4).

Insufficient information is available from which to compute the ultimate bending moment. However, if the assumption is made that the steel has a yield strength of 248 MPa (36 kip/in^2) the bending moment at which the extreme fibers of the pile will just reach yielding is at 204 m-kN (1,806 in-kip) The equivalent diameter of the section was selected as 0.43 meters (16.9 inches)..

Different boundary conditions and depths of embedment were used in the three tests. In each case, the pile head was free to rotate but the lateral load was applied at different distances above the groundline.

The tests were performed east of Paris in a fairly uniform deposit of medium-stiff clay, classified as CH by the Unified Method. The reported properties of the clay are shown in Table 1, and were found from unconfined compression and cone tests. In the absence of stress-strain curves, the value of ε_{50} was estimated and is shown in the table. The water table was below the tips of the piles, but the degree of saturation was over 90% and it is assumed that the undrained shear strength can be employed in the analyses.

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1 Kerisel, J. L., "Vertical and horizontal bearing capacity of deep foundations in clay," *Bearing Capacity and Settlement of Foundations*, Duke University, April 1965, pp. 45-51.

Of interest is that the maximum bending moment from experiment was just over 60% of that which is expected to cause yielding of the extreme fibers of the steel in the pile.

Table 1 Reported properties of soil at Bagnolet.

Depth m	Water content %	Undrained shear strength kPa	ε50*	Total unit weight kN/m3
0	--	100	0.005	17.9
3.96	31.5	125	0.005	17.9
4.69	29.0	130	0.005	17.9

*Estimated

Correlations: 1.0 kPa =20.89 lb/ft^2; 1.0 kN/m^3= 6.366 lb/ft^3

Figures 2, 3, and 4 show the results for three cases of loading. A sketch in each figure shows the pile, the ground surface, and lengths above and below the ground surface. The figures show the experimental data for the three cases, along with some correlations that were made with a given set of p-y curves. Table 2 (in pencil) show the tabulated values from the experiments of load, ground line deflection, and maximum bending moment.

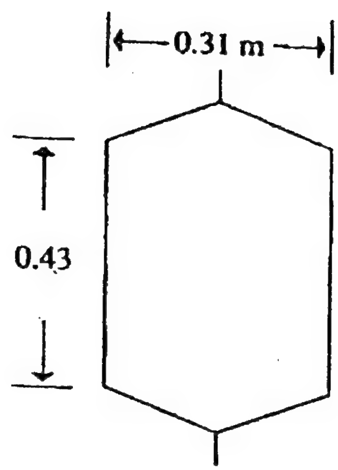
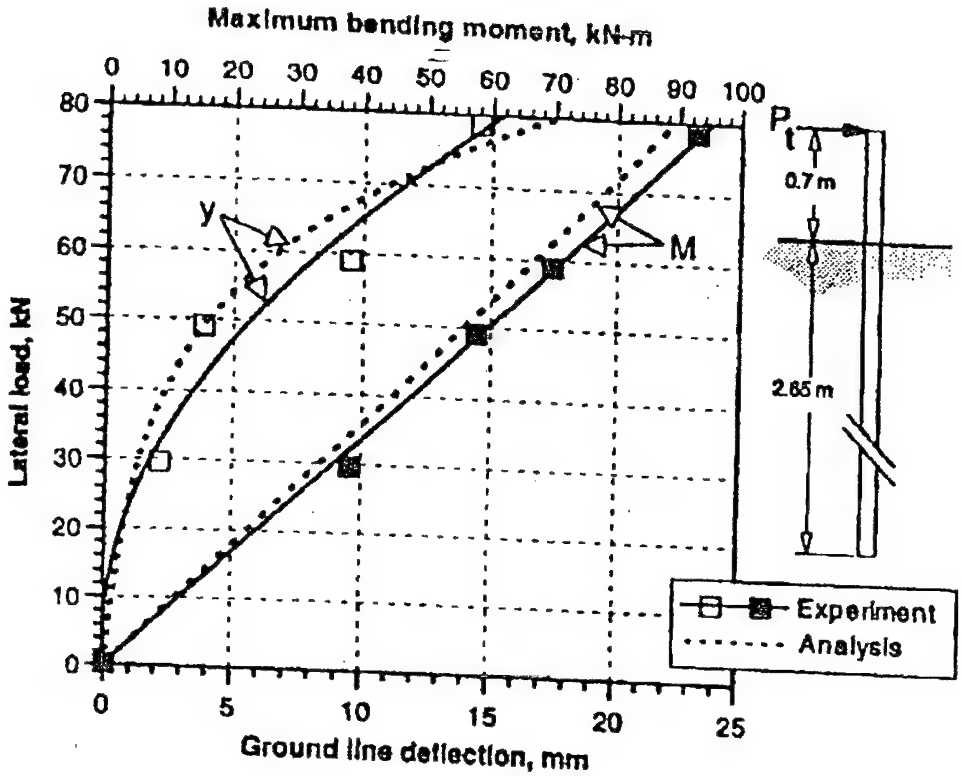


Fig. 1
Cross Section of pile at Bagnolet



Just the sketches on next 3 pages are useful. Plotting data follows in a table

Fig. 2 Comparison of Experimental and Computed Values of Maximum Bending Moment and Deflection, Case 1, Bagnolet

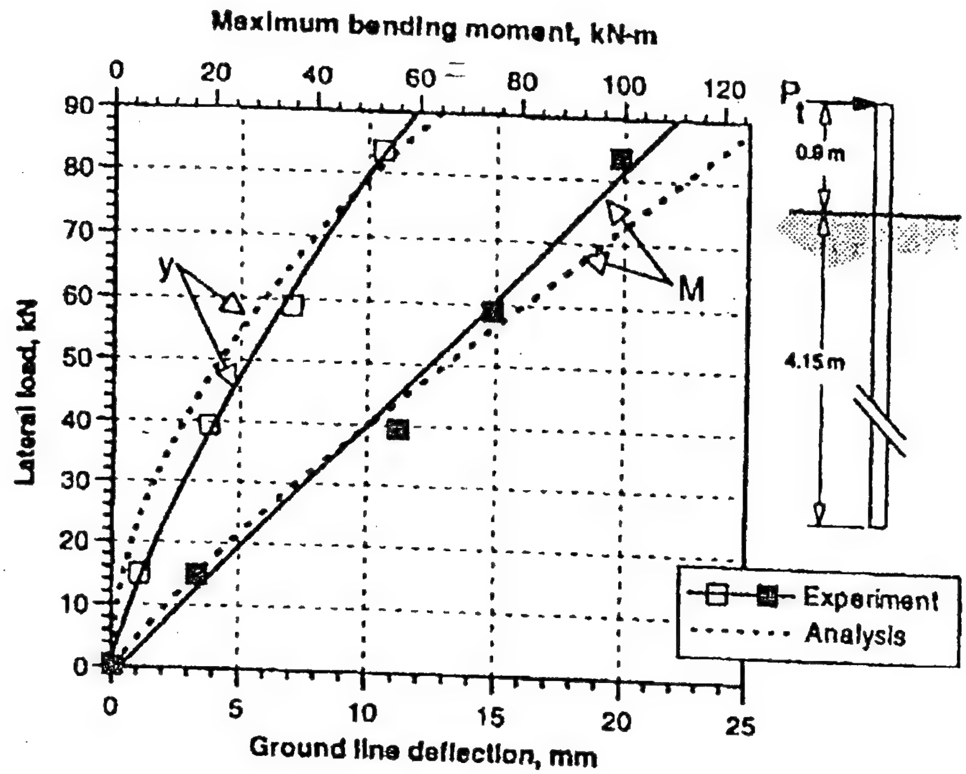


Fig. 3 Comparison of Experimental and Computed Values of Maximum Bending Moment and Deflection, Case 2, Bagnolet

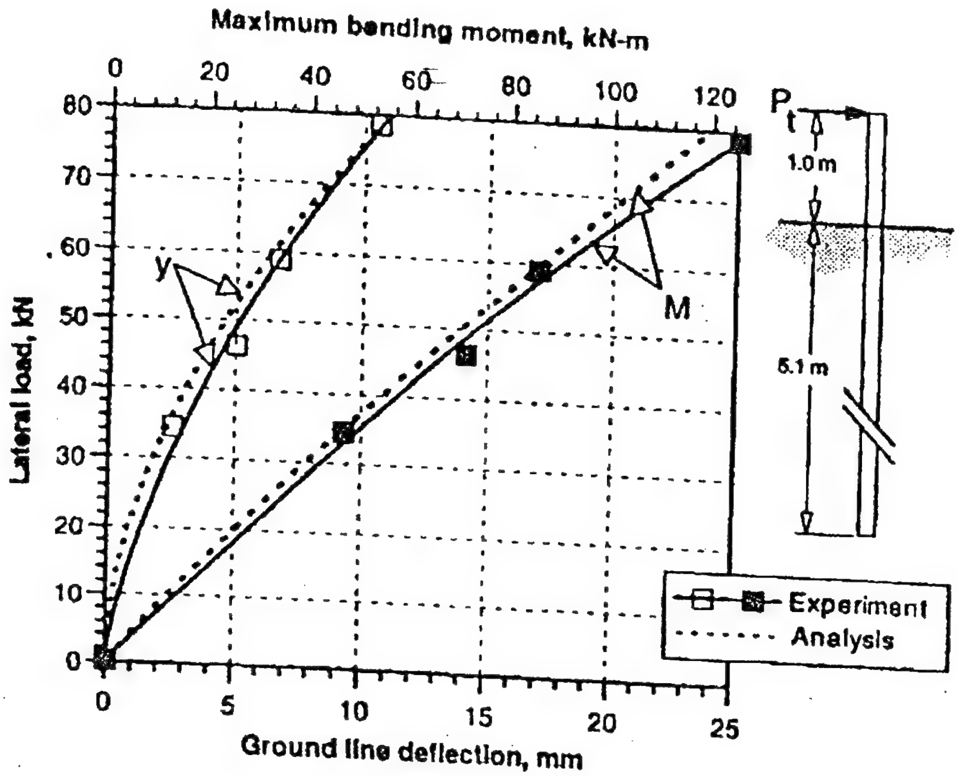


Fig. 4 Comparison of Experimental and Computed Values of Maximum Bending Moment and Deflection, Case 3, Bagnolet

BAGNOLET Table 2

Case 1

Lateral load kips	Ground line deflection in	Maximum bending moment in-kips
6.61	0.084	336
11.02	0.150	510
13.22	0.373	615
17.63	0.571	821

Case 2

Lateral load kips	Ground line deflection in	Maximum bending moment in-kips
3.30	0.041	147
8.81	0.148	494
13.22	0.275	655
18.75	0.415	876

Case 3

Lateral load kips	Ground line deflection in	Maximum bending moment in-kips
7.71	0.099	411
10.36	0.197	625
13.22	0.263	751
17.63	0.415	1106

Mustang Island (Case 3)

Cox et al. (1974)[2] describe lateral-load tests employing two steel-pipe piles that were 21 m (69ft) long. The piles were driven into sand at a site on an island near Corpus Christi, Texas. The piles were identical in design and had diameters of 0.610 m (24.0 in.). They were calibrated prior to installation and had the following mechanical properties: I = 8.0845 x 10^{-4} m^4 (1,942 in^4); EI = 163,000 kN- m^2 (56,800,000 kip-in^2); M_y = 640 kN-m (5,665 in-kip); and M_u = 828 kN-m (7,329 in-kip)[3] Both piles were instrumented internally with electrical-resistance strain gauges for the measurement of bending moment. The piles were loaded separately; Pile 1 was subjected to static loading, and Pile 2 to cyclic loading.

The soil at the site was a uniformly graded, fine sand with an angle of internal friction that was estimated to be 39 degrees based on correlations with penetration tests. Figure 1 is a plot of the data on penetration tests and relative density. Figure 2 is a plot of grain-size distribution for the sand at the site. The submerged unit weight was 10.4 kN/m^3, (66.2 lbs/ft^3); and the relative density averaged about 0.9. The water surface was maintained at 150 mm (6 in)or so above the mudline throughout the test program. The piles were driven open-ended and the modification of the sand was perhaps less than would have occurred if full-displacement piles had been installed.

The loads were applied with a hydraulic ram through a flange that was located 12 in. above the mudline. The following tables give the results of the testing.

---

[2] Cox, W. R., L. C. Reese, and B. R. Grubbs, "Field Testing of Laterally-Loaded Piles in Sand," *Proceedings*, Offshore Technology Conference, Paper No. 2079, Houston, Texas, May, 1974.

[3] M_y and M_u are the bending moments that are computed to produce a yield at the extreme fibers of the section or to develop a plastic hinge, respectively.

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Static Loading

Load, kips	Mudline deflection, in	Maximum bending moment, in-kips
0	0	0
4.8	0.03	236
7.4	0.05	379
9.9	0.07	484
12.4	0.11	664
17.4	0.17	1000
22.4	0.26	1328
27.4	0.35	1681
32.4	0.45	2071
37.2	0.56	2443
47.3	0.82	3257
54.5	1.03	3806
59.8	1.18	4284

Cyclic Loading

Load, kips	Mudline deflection, in	Maximum bending moment, in-kips
0	0	0
5.0	0.02	267
7.5	0.05	432
10.0	0.07	607
12.5	0.12	771
17.5	0.20	1130
22.5	0.28	1530
27.5	0.37	1970
32.4	0.49	2328
37.4	0.61	2823
42.4	0.77	3280
47.6	0.93	3718
54.9	1.22	4452

For static loading, a load of 59.8 kips caused the maximum bending moment of 4,284 in-kips. The computed load to cause first yield was 72.8 kips and to cause a plastic hinge was 89.0 kips. Thus, with respect to the maximum load that was applied, there were factors of safety of 1.22 and 1.49 with respect to first yield and a plastic hinge, respectively.

For cyclic loading, the maximum bending moment of 4,452 that was measured for a lateral load of 54.9 kips reflects a factor of safety of 1.2 with respect to first yield under bending, and 1.5 with respect to the ultimate moment.

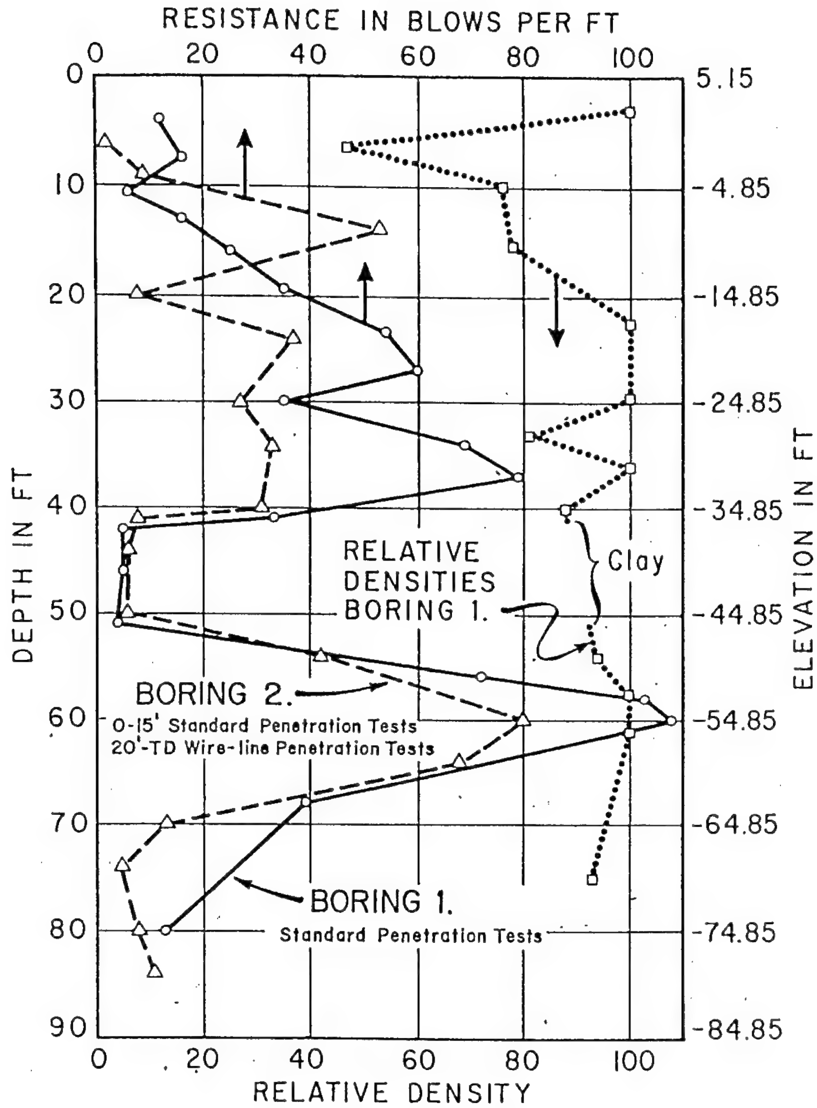

Fig. 1 - Results of standard and wire-line penetration test and relative density values from piston samples.

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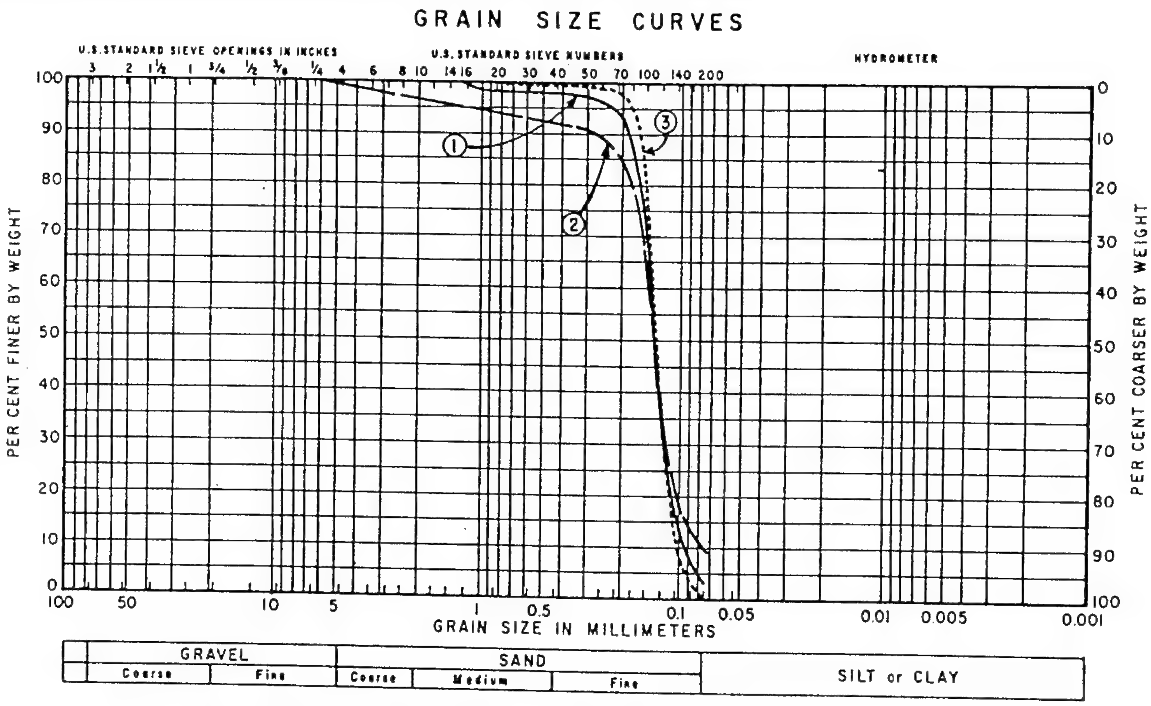


Curve No.	Boring No.	Depth, ft	Description
1	1	3	Tan fine sand with shell fragments
2	1	10	Tan fine sand with shell fragments
3	1	15.5	Light gray fine sand

(after McClelland)

Fig. 2 - Grain-size distribution curves from Boring 1, 3 to 15 1/2 ft.

Houston (Case 4)

Reese & Welch (1975)[1] reported on a test of a bored pile performed in Houston, Texas, under the sponsorship of the Texas Department of Transportation and the Federal Highway Administration. The pile had a diameter of 30 in. and a penetration of 42 feet.. An instrumented steel pipe, with a diameter of 10.75 in. and a wall thickness of 0.25 in., formed the core of the pile. A rebar cage, consisting of 20 bars, each with diameters of 1.75 in., had a diameter of 24 inches.. The yield strength of the steel was 40 kip/in^2 and the compressive strength of the concrete was 3.6 kip/in^2. The value of the bending stiffness EI was measured during the testing by reading the output from strain gauges on opposite sides on the instrumented pipe and was computed to be 1.46 x 10^8 kip-in^2. The bending moment at which a plastic hinge would occur was computed to be 18,000 in-kip.

The soil was overconsolidated clay, called Beaumont clay locally, and had a well-developed secondary structure. The water table was at a depth of 18 ft at the time of the field tests. Tube samples with a diameter of 4 in. were taken, observing the necessary precautions to reduce sampling disturbance. The properties of the clay are shown in Table 1. The undrained shear strength was measured by unconsolidated-undrained triaxial compression tests with confining pressure equal to the overburden pressure. Some samples were subjected to repeated loading and the effect on the stress-deformation relationships was observed.

The lateral loads were applied at 3 in. above the ground surface and loads were both static and cyclic. The same pile was used without redriving to obtain results for both

[1] Reese, L. C., & R. C. Welch, "Lateral loading of deep foundations in stiff clay," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 101, No. GT7, July 1975, pp. 633-649.

Table 1 Reported properties of soil at Houston site (Case 4)

Depth		Water Content	Undrained Shear Strength			Total Unit Weight	
(m)	(in)	(%)	(kPa)	(psi)	ε_{50}	(kN/m3)	(lb/ft3)
0	0.00	18.0	76.00	11.02	0.0050	19.4	123.50
0.4	15.75	18.0	76.00	11.02	0.0050	19.4	123.50
1.04	40.94	22.0	105.00	15.23	0.0050	18.8	119.68
6.1	240.16	20.0	105.00	15.23	0.0050	19.1	121.59
12.8	503.94	15.0	163.00	23.64	0.0050	19.9	126.68

The lateral loads were applied at 3 in. above the ground surface and loads were both static and cyclic. The same pile was used without redriving to obtain results for both types of loading. The successive loads were widely separated in magnitude so that the cycling at the previous load was assumed to have no effect on the first cycle at the next load. At each increment of lateral load, readings were taken at one cycle, 5 cycles, 10 cycles, and 20 cycles for the larger loads. The results from the cycling were analyzed and a method of predicting the effect of cyclic loading was developed, based on the stress level and the number of cycles.

Table 2 shows the tabulated values from the experiments for load, pile-head deflection, and maximum bending moment, for both static and cyclic loading. Also shown in Table 2 are the values of bending moment that were measured along the length of the pile for an applied load of 100 kips.

Garston (Case 5)

Price & Wardle (1987)[2] reported the results of lateral-load tests of a bored pile, identified as TP12, with a length of 41 ft and a diameter of 59.1 inches.. The location of the tests was not given and is listed as the location of the Building Research Establishment for convenience. The reinforcement consisted of 36 round bars, 1.97 in. in diameter, on a 51.2-diameter circle. The yield strength of the steel was 61.6 kip/in^2. The cube strength of the concrete was 7.22 kip/in^2. The bending moment at which a plastic hinge would occur was computed to be 141,000 in-kip at concrete strain of 0.003.

The authors installed highly precise instruments along the length of the pile. The readings allowed the determination of bending moment with considerable accuracy.

2 Price, G., & I. F. Wardle, "Horizontal load tests on steel piles in London clay," *Proceedings*, 10th ICSMFE, Stockholm, 1981, pp. 803-808.

Table 2 Tabulated values from test at Houston site (Case 4)

Experimental Results

I. STATIC LOADING

Lateral Load (kN)	Lateral Load (kips)	Pile-head Deflection (mm)	Pile-head Deflection (in)	Maximum Bending Moment (kN-m)	Maximum Bending Moment (in-kips)
0	0.00	0.0	0.00	0	0.00
89	20.01	0.5	0.02	65.9	583.26
177.9	39.99	2.3	0.09	150.3	1330.27
266.9	60.00	6.5	0.26	270	2389.70
355.8	79.99	14.9	0.59	438.4	3880.17
444.8	100.00	29.5	1.16	627	5549.42

II. CYCLIC LOADING

Lateral Load (kN)	Lateral Load (kips)	Pile-head Deflection (mm)	Pile-head Deflection (in)	Maximum Bending Moment (kN-m)	Maximum Bending Moment (in-kips)
0	0.00	0.0	0.00	0	0.00
89	20.01	0.5	0.02	65.5	579.72
177.9	39.99	3.0	0.12	161.6	1430.28
266.9	60.00	9.9	0.39	313	2770.28
355.8	79.99	22.1	0.87	505	4469.63
444.8	100.00	39.6	1.56	703.9	6230.04

III. MOMENT -vs- DEPTH

Depth (m)	Depth (in)	Bending Moment (kN-m)	Bending Moment (in-kips)
0	0.00	0.0	0.00
0.5	19.69	205.2	1816.17
1	39.37	387.5	3429.66
1.5	59.06	502.2	4444.84
2	78.74	578.1	5116.62
2.5	98.43	617.9	5468.88
3	118.11	624.5	5527.29
3.5	137.80	609.0	5390.10
4	157.48	575.9	5097.14
5	196.85	485.5	4297.04
6	236.22	333.2	2949.07
7	275.59	187.6	1660.40
8	314.96	83.9	742.58
9	354.33	17.7	156.66

The properties of soil reported by the authors are shown in Table 3. Also shown in the table are estimated values of the angle of internal friction.

The lateral load was applied at 35.4 in. above the ground line. Each load was held until the rate of movement was less than 0.05 mm in 30 minutes. The load was reduced to zero in stages and held at zero for one hour.

Table 4 shows the tabulated values from the experiments for load, pile-head deflection, and maximum bending moment.

Table 3 Reported properties of soil at Garston site (Case 5)

Depth				Total Unit Weight		Friction Angle
(m)	(in)	Description	Nspt	(kN/m3)	(lb/ft3)	(degrees)
0 - 0.36	0 - 14	Fill	18	---	---	---
0.36 - 3.5	14 - 138	Dense sandy gravel	~65	21.5	136.87	43
3.5 - 6.5	138 - 256	Coarse sand and gravel	30	9.7	61.75	37
6.5 - 9.5	256 - 374	Weakly cemented sandstone	~61	11.7	74.48	43
9.5 -	374 -	Highly weathered sandstone	~140	---	---	---

Table 4 Tabulated values from test at Garston site (Case 5)

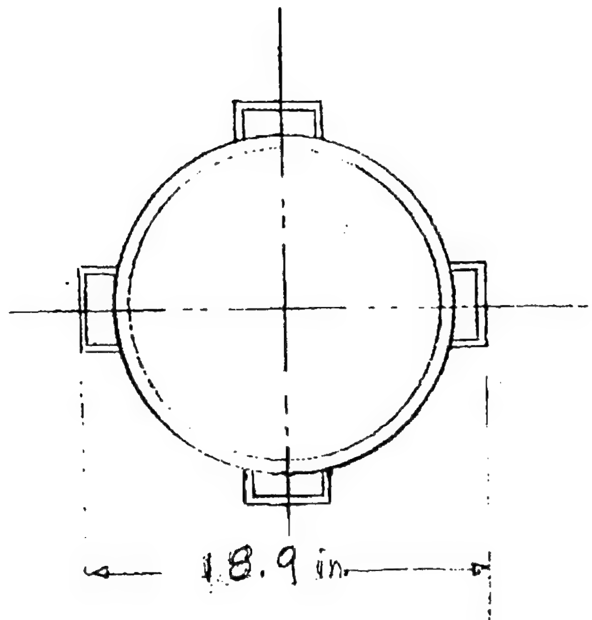
Experimental Results

I. STATIC LOADING

Lateral Load (kN)	Lateral Load (kips)	Pile-head Deflection (mm)	Pile-head Deflection (in)
0	0.00	0.0	0.00
200	44.96	0.8	0.03
400	89.92	2.5	0.10
600	134.89	4.5	0.18
800	179.85	7.1	0.28
1000	224.81	10.5	0.41
1200	269.77	15.7	0.62
1400	314.73	20.5	0.81
1600	359.70	25.9	1.02
1800	404.66	32.7	1.29
2000	449.62	38.0	1.49
2200	494.58	46.1	1.81
2400	539.56	51.3	2.02

II. LATERAL LOAD -vs- MAX. BENDING MOMENT

Lateral Load (kN)	Lateral Load (kips)	Maximum Bending Moment (kN-m)	Maximum Bending Moment (in-kips)
0	0.00	0.0	0.00
600	134.89	1410.0	12479.55
2400	539.54	8800.0	77886.56



Note for drafting:

Pipe OD = 16.0 in.

t = 0.32 in

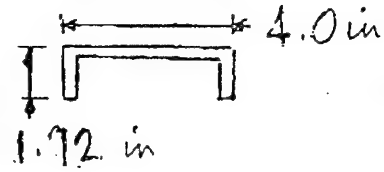


Fig. 1

Cross Section of Pile at Arkansas River

Table 1

Depth, ft	N_{SPT}
0	12
2	12
7.9	14
13	20
15	17
18	25
23	28
28	18
33	27
38	29
65.6	29 (estimated)

The values of N_{SPT} in the table were used to estimate the angle of internal friction, based on the paper by Gibbs & Holtz (1957)[3], and the initial stiffnesses of the p-y curves at the site. The correlation indicated an average value of ϕ of 40 degrees. However, the site had been preconsolicated because about 20 ft of overburden was removed prior to testing; further, there could have been densification of the soil due to the installation of the pile; therefore, the value of ϕ may have as high a value as 45 degrees.

Table 2 shows the resullts of the load test. The loads were applied at the ground line and the deflection was measured at the same point.

[3] Gibbs, H. J., & W. G. Holtz, "Research on determining the density of sands by spoon penetration testing," *Proceedings,* Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957, Vol. 1, pp. 35-39.

TABLE 2

Lateral load, kips	Pile-head Deflection, in.
0	0
10.5	0.07
21	0.17
31.5	0.27
43	0.39
56	0.55

Brent Cross (Case 7)

Price & Wardle (1981[4]) reported the results of a test of a steel pipe in London Clay. The diameter of the pile was 16.0 in., the wall thickness was 0.395 in., and its penetration was 54 feet.. In addition to the analysis of the test by the original authors, Gabr et al. (1994)[5] did a further study. The moment of inertia, *I*, of the pile was reported as 588 in^4; the bending stiffness, *EI*, used in the analyses 17,900,000 kip-in^2. The bending moment at which the extreme fibers would reach yield was computed to be 2,664 in-kip, and the ultimate bending moment, at which a plastic hinge would develop, was computed to be 3,470 in-kip.

The data on the properties of the London Clay at the site was obtained from the testing of specimens taken with thin-walled tubes with a diameter of 3.86 inches. The water table was presumably at some distance below the ground surface. The values, shown in Table 3, of undrained shear strength were scaled from a plot presented by the authors.

TABLE 3

Depth, ft	Undrained shear strength,kip/in^2
0	6.40
15.1	12.4
20.3	11.7
62.3	19.3

Data on the stiffness of the soil were not reported and values of ε_{50} were estimated to range from 0.01 to 0.005, which should be in the range of the values for London Clay.

[4] Price, G., & I. F. Wardle, "Horizontal load tests on steel piles in London clay," *Proceedings*, 10th ICSMFE, Stockholm, 1981, pp. 803-808.

[5] Gabr, M. A., T. Lunne, & J. J. Powell, "P-y analysis of laterally loaded piles in clay using DMT," *Journal of Geotechnical Engineering*, ASCE, Vol. 120, No. 5, May 1994, pp. 816-837.

The lateral load was applied at 1.0 m above the groundline and both static and cyclic loads were applied. The static loads were of a larger magnitude than the cyclic loads and were applied after cycling had been done. The assumption is made that the cycling with the smaller loads did not affect the subsequent static results. The deflection was measured at the point of the application of the load. Only the results from the static loading are reported here.. Table 4 shows values of applied lateral load and the resulting deflection.

TABLE 4

Load, kips	Deplection at point of load application, in.
0	0
4.5	0.09
9.0	0.21
13.5	0.46
22.5	1.29

Japan (Case 8)

The Committee on Piles Subjected to Earthquake (1965)[1] reported the results from the testing of a steel-pipe pile with a closed end that was jacked into the soil. The pile was 12-in in outside diameter with a wall thickness of 0.125-in, and its penetration was 17-ft. The moment of inertia, I was 82.4-in^4, and the bending stiffness, EI, was 2,393x10^6 kip-in^2. The bending moment My at which yielding of the extreme fibers would occur was computed to be 495 in-kip, and the ultimate bending moment Mu was computed to be 636 in-kip.

The soil at the site was a soft, medium to highly plastic, silty clay with a high sensitivity. The undrained shear strength and stiffness of the soil was obtained from undrained triaxial shear tests. The strains at failure were generally less that 5%, and failure was by brittle fracture. The properties of the soil are shown in Table 1.

The loading was applied at 7.91-in above the groundline; the maximum lateral load was a moderate value of 3.20-kips which produced a pile head deflection of 0.19-in and a maximum bending moment of 153.5 in-kip. Thus, with respect to a failure due to yielding of the extreme fibers, the factor of safety was 3.2 and the factor of safety against a failure in plastic yielding was 4.1.

The loading was static. Table 2 shows the values of the experimental pile-head deflection as a function of the pile-head lateral load and the values of maximum bending moment as a function of pile-head lateral load.

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[1] Committee on Piles Subjected to Earthquake, Architectural Institute of Japan, Lateral Bearing Capacity and Dynamic Behavior of Pile Foundation (Loading Test of Single and Grouped Piles), May, 1965, pp. 1-69 (in Japanese).

Bending moments were measured at the site but information is unavailable on the techniques that were used.

Even though the length to diameter ratio was relatively small at 17, examination of the results for deflection and bending moment along the pile showed that the pile would have failed in bending rather than by excessive deflection.

Table 1 Reported properties of soil at Japan site (Case 8)

Depth		Undrained Shear Strength			Submerged Unit Weight	
(m)	(ft)	(kPa)	(psf)	ε50	(kN/m3)	(lb/ft3)
0	0	27.30	570	0.0050	4.9	31
5.18	17	43.10	900	0.0050	4.9	31

Table 2 Tabulated values from test at Japan site (Case 8)

Lateral Load		Pile-head Deflection		Maximum Bending Moment	
(kN)	(kips)	(mm)	(in)	(kN-m)	(in-kips)
0	0.00	0.00	0.00	0	0.00
3.55	0.80	1.01	0.04	4.3	38.06
8.76	1.97	2.56	0.10	8.67	76.74
14.24	3.20	4.83	0.19	17.34	153.47

Alcácer do Sol (Case 9)

Portugal & Sêco e Pinto (1993)[1] describe the testing of a bored pile at the site of a bridge at Alcácer do Sol. Three piles were tested and the results for Pile 2 are shown here.

The pile was 131-ft in length and had a diameter of 47.2-in. It was reinforced with 35 bars with a diameter of 1.0-in. The strengths of the concrete and steel were reported to be 4,860-psi and 5,800-psi, respectively. The cover of the rebars was taken as 2.0-in. The bending stiffness was computed, and a value of 1.146×10^9 kip-in^2 was selected for use in the analyses. The ultimate bending moment was computed to be 29,800 in-kip. The pile was instrumented for the measurement of bending moment along its length.

From the ground surface downward, the soil is described as silty mud, sand, muddy complex, and sandy complex. The properties of the soil were found from SPT, CPT, and vane tests, and the values that were selected for use in the analyses are shown in Table 1.

The lateral load was applied at 7.9-in above the ground line. Bending moment was measured along the length of the pile but information is unavailable on the techniques that were used. Ground-line deflection and maximum bending moment were reported for three values of lateral load: 22.5, 45, and 67.5 kips.

[1] Portugal, J. C., & P. S. Sêco e Pinto, "Analysis and design of piles under lateral loads," Deep Foundations on Bored and Auger Piles, BAP II, *Proceedings of the 2nd International Geotechnical Seminar on Deep Foundations on Bored and Auger Piles*, Ghent, Belgium, 1993, pp. 309-313.

The position of the water table was not reported but it is assumed that the water table was close to the ground surface. Table 2 shows the experimental results of pile head deflection and maximum bending moment as a function of lateral load.

Table 1 Reported properties of soil at Alcácer do Sol site (Case 9)

Depth		Undrained Shear Strength			Total Unit Weight	
(m)	(ft)	(kPa)	(psf)	ε50	(kN/m3)	(lb/ft3)
0	0	20.00	418	0.0100	16	102
3.5	11	20.00	418	0.0100	16	102
3.5	11	-------	-------	-------	19	121
8.5	28	-------	-------	-------	19	121
8.5	28	32.00	668	0.0100	16	102
23	75	32.00	668	0.0100	16	102
23	75	-------	-------	-------	19	121
40	131	-------	-------	-------	19	121

Table 2 Tabulated values from test at Alcácer do Sol site (Case 9)

Lateral Load		Pile-head Deflection		Maximum Bending Moment	
(kN)	(kips)	(mm)	(in)	(kN-m)	(in-kips)
0	0.00	0.00	0.00	0	0.00
100	22.48	1.80	0.07	270	2389.70
200	44.96	4.50	0.18	583	5159.98
300	67.44	8.10	0.32	1007	8912.70

Lake Austin (Case 10)

Matlock (1970)[1] presented results from lateral-load tests employing a steel-pipe pile that was 12.75-in in diameter, with a wall thickness of 0.50-in, and a length of 42-ft. The bending moment at which the extreme fibers would first yield was computed to be 2,045 in-kip.

The pile was driven into clays near Lake Austin, Texas, that were slightly overconsolidated by desiccation, slightly fissured, and classified as CH according to the Unified System. The undrained shear strength was measured with a field vane; was found to be almost constant with depth, and averaged 800 lb/ft^2.

A comprehensive investigation of the soil was undertaken and the computations shown herein are based on tests with the field vane. The vane strengths were modified to obtain the undrained shear strength of the clay. The values of the soil properties employed in the following computations are shown in Table 1. The value of ε_{50} was found from triaxial tests and averaged 0.012. The submerged unit weight was 63.7 lb/ft^3, and water was kept above the ground surface during all of the testing.

The pile was tested under static loading, removed, redriven, and tested under cyclic loading. The load was applied at 2.5-in above the groundline.

The pile was instrumented internally, at close spacings, with electrical-resistance strain gauges for the measurement of bending moment. Each increment of load was allowed to remain long enough for readings of strain gauges to be taken by an extremely

[1] Matlock, H., "Correlations for design of laterally loaded piles in soft clay," *Proceedings*, Second Annual Offshore Technology Conference, Houston, 1970, Vol. 1, pp. 577-588.

precise device. A rough balance of the external Wheatstone bridge was obtained by use of a precision decade box and the final balance was taken by rotating a drum, 6-in in diameter, on which a copper wire had been wound into a spiral groove in the drum. A contact on the copper wire was read on the calibrated drum when a final balance was achieved. The accuracy of the device was less than one microstrain but, unfortunately, some time was required for readings to be taken from the top of the pile to the bottom and back up again. Because of the creep of the soil at relatively moderate to high loadings, the pressure in the hydraulic ram that controlled the load was adjusted as necessary to maintain a constant load. The two sets of readings at each point along the pile were interpreted to find that reading at a particular time, assuming that the change in moment due to creep, had a constant rate.

Table 2 shows the experimental results of pile-head deflection as a function of lateral load and maximum bending moment as a function of lateral load.

A lateral load of 18.2-kips caused a maximum bending moment of approximately one-half of the 2,045 in-kip that would cause the first yield. Table 3 shows the experimental values of bending moment as a function of depth.

Table 1 Reported properties of soil at Lake Austin site (Case 10)

Depth (ft)	Water Content* (%)	Undrained Shear Strength (psf)		
0	29.0	631		
3.7	33.5	673		
3.7	33.5	883		
11.1	50.1	366		
12.1	49.6	629		
14.1	48.3	489		
18.7	46.1	1082		
23.8	54.5	622		
31.1	55.5	681		
49.2		681		

* Average values.

Table 2 Tabulated values from test at Lake Austin site (Case 10)					
Lateral Load		Pile-head Deflection		Maximum Bending Moment	
(kN)	(kips)	(mm)	(in)	(kN-m)	(in-kips)
0	0.00	0.00	0.00	0	0.00
9.61	2.16	2.40	0.09	12.29	108.78
20.99	4.72	6.90	0.27	26.69	236.23
32.96	7.41	11.80	0.46	43.74	387.13
44.97	10.11	16.40	0.65	58.3	516.00
56.93	12.80	21.60	0.85	73.05	646.55
68.94	15.50	27.80	1.09	88.97	787.45
80.91	18.19	35.00	1.38	107.06	947.56
92.92	20.89	43.50	1.71	124.19	1099.17
104.88	23.58	56.10	2.21	150.43	1331.42

Table 3 Moment -vs- Depth, Lake Austin (Case 10)			
Depth		Maximum Bending Moment	
(m)	(ft)	(kN-m)	(in-kips)
0.00	0.00	0	0.00
0.305	1.00	23.8	210.65
0.61	2.00	43.20	382.35
0.914	3.00	61.10	540.78
1.219	4.00	75.90	671.77
1.524	5.00	87.70	776.21
1.829	6.00	97.00	858.52
2.134	7.00	100.60	890.39
2.438	8.00	103.40	915.17
2.667	8.75	106.80	945.26
2.743	9.00	106.80	945.26
3.048	10.00	101.90	901.89
3.353	11.00	96.80	856.75
3.658	12.00	89.00	787.72
3.962	13.00	76.60	677.97
4.572	15.00	58.30	516.00
5.182	17.00	32.90	291.19
5.486	18.00	22.30	197.37
6.096	20.00	7.30	64.61
6.706	22.00	0.00	0.00

Sabine (Case 11)

The pile tested at Lake Austin was removed and installed at Sabine where the soil was a soft clay (Matlock, 1970)[1]. As before, the pile was tested both under static and cyclic loading. Meyer (1979)[2] analyzed the results of the Sabine tests and reported that the clay was a slightly overconsolidated marine deposit, had an undrained shear strength of 300 lb/ft^2, and a submerged unit weight of 35 lb/ft^3. Computations were made with values of ε_{50} of 0.01 and 0.02.

The lateral loads were applied at 12-in above the ground line. Table 1 shows the experimental results of pile head deflection and maximum bending moment as a function of lateral load.

Table 1 Tabulated values from test at Sabine site (Case 11)

Lateral Load		Pile-head Deflection		Maximum Bending Moment	
(kN)	(kips)	(mm)	(in)	(kN-m)	(in-kips)
0	0.00	0.00	0.00	0	0.00
18.4	4.14	6.00	0.24	23.8	210.65
35.8	8.05	17.40	0.69	54.9	485.91
52.8	11.87	33.90	1.33	89.5	792.14
70.7	15.89	57.10	2.25	132.2	1170.07
80.1	18.01	71.60	2.82	156.50	1385.14

[1] Matlock, H., "Correlations for design of laterally loaded piles in soft clay," *Proceedings*, Second Annual Offshore Technology Conference, Houston, 1970, Vol. 1, pp. 577-588.

[2] Meyer, B. J., "Analysis of single piles under lateral loading," Thesis, The University of Texas at Austin, unpublished, 1979.

Manor (Case 12)

Reese et al. (1975)[1] describe lateral-load tests employing two steel-pipe piles that were 50-ft long, with a diameter of the upper section of 25.2-in and of the lower of 24.0-in. The piles were driven into stiff clay at a site near Manor, Texas. The piles were calibrated prior to installation and the mechanical properties of each of the piles are shown in Table 1. The bending moment, M_y, when yield stress develops at the extreme fibers and the ultimate bending moment, M_u, are shown only for the top sections, where the ultimate bending moment occurs during loading.

The clay at the site was strongly overconsolidated and there was a well developed secondary structure. The undrained shear strength of the clay was measured by unconsolidated-undrained consolidation tests with confining pressure equal to the overburden pressure. The properties of the clay are shown in Table 2.

Both of the piles were instrumented with electrical-resistance strain gauges for measurement of bending moment. The gauge-readings were taken with an electronic data-acquisition system, and a full set of readings could be taken in about one minute. The point of application of the load for both piles was 12-in above the groundline. Pile 1 was tested under static loading with the load being increased in increments; loading ceased when the bending moment was near the yield moment.

Pile 2 was tested under cyclic loading and the loads were cycled under each increment until deflections were stabilized. The number of cycles of loading was in the order of 100 and applied at a rate of about two cycles per minute. Observations during

and after testing revealed that the erosion was significant in response to the cyclic loading. A gap was revealed in front of the pile after removing a load, except for the loads of very low magnitude; the gap became filled with water that was ejected during the next cycle of loading. The water was caused to rush upward at a high velocity and carried particles of clay. During the testing, radial cracks developed in the ground surface in front of the pile and a subsequent examination showed that erosion has occurred outward through the cracks as well as in front of the pile.

While the equilibrium condition was reached with about 100 cycles of loading, as noted above, it is probable that the application of hundreds or thousands of cycles would have caused additional deflection. The question of the expected number of cycles of loading, particularly for clay soils below free water, needs careful attention in any problem of design. Also, the gapping around a pile is plainly related to the loss of resistance during cyclic loading, and gapping may be related to diameter other than by the first power as implied by the recommendations for *p-y* curves.

For static loading, Table 3 shows the experimental results of pile head deflection and maximum bending moment as a function of lateral load. The maximum bending moment of 11,240 in-kip that was measured reflects a factor of safety of 1.38 with respect to the yield moment and 1.83 with respect to the ultimate moment.

Table 1 Reported properties of piles at Manor site (Case 12)

PILE	Depth (ft)	Moment of Inertia (in4)	Flural Stiffness EI (kip-in2)	Yielding Moment My (in-kip)	Ultimate Moment Mu (in-kip)
1	Top 23 ft	5,610	1.72E+08	15,550	20,550
	Bottom 27 ft	--------	5.87E+07	--------	--------
2	Top 23 ft	5,610	1.67E+08	15,550	20,550
	Bottom 27 ft	--------	6.08E+07	--------	--------

[1] Reese, L. C., & R. C. Welch, "Lateral loading of deep foundations in stiff clay," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 101, No. GT7, July 1975, pp. 633-649.

Table 2 Reported properties of soil at Manor site (Case 12)				
Depth (ft)	Water Content* (%)	Undrained Shear Strength (psf)	ε50*	Total Unit Weight (pcf)
0	---	522	0.005	---
3	37	1,460	0.005	115
5	27	3,400	0.005	124
13.5	22	6,960	0.005	129
21.5	22	6,960	0.005	129
30	19	23,000	0.005	132
66	---	23,000	0.005	---
* Average values.				

Table 3 Tabulated values of test at Manor site (Case 12)					
Lateral Load		Pile-head Deflection		Maximum Bending Moment	
(kN)	(kips)	(mm)	(in)	(kN-m)	(in-kips)
0	0.00	0.00	0.00	0.0	0.00
37.9	8.52	0.30	0.01	45.2	400.05
63.7	14.32	0.70	0.03	81.3	719.57
108.6	24.41	1.60	0.06	152.5	1,349.74
130.2	29.27	2.10	0.08	186.4	1,649.78
180.1	40.49	3.10	0.12	267.8	2,370.23
200.1	44.98	3.60	0.14	303.9	2,689.74
223.7	50.29	4.20	0.17	348.0	3,080.06
266.4	59.89	5.40	0.21	451.6	3,997.00
317.7	71.42	7.00	0.28	532.1	4,709.48
356.5	80.14	8.40	0.33	616.9	5,460.03
405.8	91.23	10.10	0.40	717.4	6,349.52
449.3	101.01	12.70	0.50	926.4	8,199.33
485.6	109.17	14.20	0.56	1,091.0	9,656.16
538.1	120.97	17.80	0.70	1,175.0	10,399.63
578.5	130.05	19.70	0.78	1,227.0	10,859.86
606.2	136.28	21.70	0.85	1,271.0	11,249.30

Florida (Case 13)

Davis (1977)[1] described the testing of a steel-pipe pile that had a diameter 4.66-ft and a penetration of 26-ft. The tube was filled with concrete to a depth of 4.0-ft, and a utility pole was embedded so that the lateral loads were applied at 51-ft above the ground line. Meyer (1979)[2] analyzed the results of the test and reported the bending stiffness to be 1.77×10^9 kip-in^2 in the top 4.0-ft, and 8.80×10^8 kip-in^2 below. The ultimate bending moment was reported to be 55,600 in-kip in the top 4.0-ft, and 39,000 in-kip in the lower portion.

The soil profile consisted of 13-ft of sand above saturated clay. The sand had a total unit weight of 122 lb/ft^3, and an angle of internal friction of 38 degrees. The water table was at a depth of 2-ft. The undrained shear strength of the clay was 2,500 lb/ft^2, and its submerged unit weight was 60 lb/ft^3. A value of ε_{50} of 0.01 was selected for the analyses.

Preliminary analyses showed that the bottom of the relatively short pile was deflecting. Table 1 shows lateral resistance of the soil at the base of the pile as a function of lateral deflection. The table was developed by making use of recommendations for load transfer in skin friction for piles under axial load. The experimental results, showing pile-head deflection as a function of the lateral load at the pile head, are shown in Table 2.

[1] Davis, L. H., "Tubular steel foundation," Test Report RD-1517, Florida Power and Light Company, Miami, Florida, 1977.
[2] Meyer, B. J., "Analysis of single piles under lateral loading," Thesis, The University of Texas at Austin, unpublished, 1979.

Table 1 Computed force-displacement relationship for lateral deflection of the tip of the pile, Florida site (Case 13)				
	Force	**Displacement**		
	(kips)	**(in)**		
	0.0	0.000		
	17.0	0.078		
	26.5	0.157		
	32.1	0.236		
	36.9	0.315		
	40.0	0.394		
	41.1	0.472		
	42.3	0.551		
	42.7	0.590		
	42.7			

Table 2 Tabulated values from test at Florida site (Case 13)				
Lateral Load		**Pile-head Deflection**		
(kN)	**(kips)**	**(mm)**	**(in)**	
0	0.00	0.00	0.00	
46	10.34	1.50	0.06	
68	15.29	3.20	0.13	
89	20.01	5.10	0.20	
113	25.40	7.80	0.31	
133	29.90	10.20	0.40	
158	35.52	11.40	0.45	
182	40.92	15.00	0.59	
226	50.81	19.50	0.77	

Apapa (Case 14)

Coleman (1968)[1], and Coleman & Hancock (1972)[2] describe the testing of Raymond step-tapered piles near Apapa, Nigeria. The results were analyzed by Meyer (1979)[3]. Two piles, identical in geometry, were driven and capped with concrete blocks. A hydraulic ram was placed between the caps and the piles were loaded by being pushed apart. The reported properties of the piles are presented in Table 1.

The soil at the site consisted of 5-ft of dense sand underlain by a thick stratum of soft organic clay. The angle of internal friction of the sand was obtained in the laboratory by triaxial tests of reconstituted specimens. The strength of the soft organic clay was obtained from in situ-vane tests. The water table was at a depth of 3-ft. Table 2 shows the properties of the soil that were reported.

The lateral loads were applied at 2-ft above the ground line, and the deflection was measured at that point. The experimental results are shown in Table 3 for both piles. The pile-head deflection is shown as a function of the lateral load.

[1] Coleman, R. B., "Apapa road Ijora causeway reconstruction," Report on Horizontal Load Tests on Piles, Federal Ministry of Works and Housing, Ijora, Lagos, Nigeria, January 1968.
[2] Coleman, R. B., & T. G. Hancock, "The behavior of laterally loaded piles," *Proceedings*, Fifth European Conference on Soil Mechanics and Foundation Engineering, Madrid, Vol. 1, 1972, pp. 339-345.
[3] Meyer, B. J., "Analysis of single piles under lateral loading," Thesis, The University of Texas at Austin, unpublished, 1979.

Table 1 Mechanical properties of piles at Apapa site (Case 14)				
			Flexural	
	Section	Diameter	Stiffness	
	(ft)	(in)	(kip-in2)	
	0 - 8	17.4	7.80E+06	
	8 - 20	16.4	7.00E+06	
	20 - 50	15.4	6.50E+06	

Table 2 Soil properties at Apapa site (Case 14)					
			Undrained		
Depth		Phi Angle	Shear Strength	Unit Weight	ε50
(ft)	Soil Type	(degrees)	(psf)	(lb/ft2)	
0 - 3	Sand Fill	41.0	----	120	----
3 - 5	Sand Fill	41.0	----	68	----
5 - 20	Peat and Soft Clay	----	500.0	30	0.02

Table 3 Tabulated values from test at Apapa site (Case 14)					
		PILE 1		PILE 2	
Lateral Load		Pile-head Deflection		Pile-head Deflection	
(kN)	(kips)	(mm)	(in)	(mm)	(in)
0	0.00	0.00	0.00	0.00	0.00
8.9	2.00	1.50	0.06	1.30	0.05
17.8	4.00	3.20	0.13	3.20	0.13
26.7	6.00	4.40	0.17	5.10	0.20
35.6	8.00	5.80	0.23	6.80	0.27
44.5	10.00	7.40	0.29	8.80	0.35
53.3	11.98	9.60	0.38	12.90	0.51
62.2	13.98	13.50	0.53	18.60	0.73
71.2	16.01	18.30	0.72	24.60	0.97
80.1	18.01	22.80	0.90		
89	20.01	27.40	1.08		